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Discussion of "FLOOD INSURANCE"

by H. Alden Foster
(Proc. Sep. 483)

H. ALDEN FOSTER,[1] M. ASCE.—The discussers have emphasized certain matters connected with flood insurance which are intended to point the way towards possible cooperative action by governmental agencies and the insurance companies. While the writer is not in a position to know whether the companies would be willing to participate in such an arrangement, he believes the inherent problems are such as to render any such cooperative action most unlikely.

Mr. Edgar E. Foster's recommendation for elimination of Indirect Losses and Depreciation Losses is in line with the writer's suggestion in the paper. Mr. Foster believes that emphasis should be placed on "annual flood risk" rather than "annual average loss." It would seem that these are substantially the same concept. Loss implies past experience; while "risk" considers future conditions which, however, are estimated by reference to the past records.

Mr. Foster is naturally much interested in the most suitable method for determining flood probabilities, and he points out the particular advantages of Prof. J. J. Slade's partly bounded function. The writer intentionally omitted any discussion of the relative advantages of different methods of probability analysis as applied to floods, since the discrepancies between the various methods which have received considerable attention by hydrologists are overshadowed by the practical difficulties resulting from errors of sampling and from the importance of spreading the risk. Since it is doubtful whether any theoretical method of analysis would receive universal acceptance by the profession, the writer feels that a pragmatic approach is justified.

High insurance rates should obviously be an indication of the high risks involved in occupying areas subject to frequent floods. But that would not give much assurance that the property owners would be willing to take out the necessary insurance to protect their interests.

The writer believes there are fundamental differences in the functions of life insurance as compared with fire insurance and flood insurance. Life insurance is intended to protect the policy-holder against the risk of premature death or death prior to the time that a person has been able to accumulate a degree of financial competence. The person who lives beyond his allotted "three score years and ten" is assumed to be so pleased that he will agree in advance to contribute to the assistance of the beneficiaries of other policyholders who are less fortunate. The risk of loss resulting from premature death is thus spread among all the policy holders.

With fire insurance there is even more spreading of the risk, for it is not certain that every property will eventually be damaged by fire. As a result, many owners of property who never suffer any fire loss contribute to the

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reimbursement of the relatively few policy-holders whose property is damaged in any year.

With flood insurance, on the other hand, it is certain that all properties located in the flood plain of an uncontrolled river will be damaged some time. Those properties subjected to damage by a flood equal to the annual flood, with anticipated occurrence of at least once every year, are certain to be damaged every year. In this case there is actually no risk involved, as the occurrence of the flood event is certain. Where there is no risk there can be no function for insurance, since insurance is fundamentally the assumption of risk. In the case of damage by the annual flood, the annual insurance premium to cover such damage would have to be equal to the annual loss plus the loading charge. In other words, the annual insurance premium would exceed the annual damage to the property, and the owner would be better off without any insurance "protection."

As pointed out in the paper, the only way in which to spread the risk of flood losses is by having the policies of any single company scattered over a large area so that only a portion of all the policies would become payable in any given year. This would reduce the total amount of reserves that the company would have to maintain, but would not reduce the average annual loss which directly controls the premium rates on the outstanding policies. Providing for Governmental assistance by re-insurance would not spread the risk but would spread the cost because taxpayers all over the country would be contributing to the reimbursement of policy-holders who have suffered damage due to a disastrous flood.

The suggestion by Mr. Smith and Professor Posey, for establishing a limiting flood stage to define the "minimum deductible clause," might have a beneficial effect by making it clear that the property owner would save money by providing a moderate degree of flood protection at his own expense. The diagram submitted by Mr. Smith brings out clearly the relation between flood insurance and flood control construction, and shows that the optimum expenditure for protection when combined with insurance is reached when the differential between construction benefits and costs is a maximum. However, such a combination of government aid by flood control measures and private insurance would not eliminate the basic problems that the insurance company would have to face,—spreading the risk and providing adequate capital reserves,—though the financial magnitude of these problems would be reduced.

The combination of flood protection works and insurance might be developed by combined action of the government and the insurance companies, provided the companies were willing to undertake such business. However, after partial flood protection has been provided by government-financed construction, the insurance company might have difficulty in inducing the property owners to participate in an insurance program, due to over-confidence in the protection provided by the flood-control works.

The basic difficulties involved in setting up a program of flood insurance would not be eliminated by having Government take over the risks of the business. This would merely force the public in general to assume the losses caused by flooding instead of letting the individual property owners share these losses among themselves, as is done with other types of insurance.

In summary, the writer would emphasize that the purpose of the paper was primarily to develop the engineering problems that would be involved in setting up an insurance program. Numerous other factors would also be involved which are primarily related to technicalities of the insurance business, and outside the scope of the paper and the experience of the writer.

Examination of the engineering problems reveals fundamental difficulties which render the peril of flood uninsurable from a commercial standpoint. It does not appear that these difficulties can be eliminated by requiring Government to take responsibility for issuing such insurance policies. Floods are caused by the forces of Nature, and turning over to Government the job of protecting property owners against loss does not remove Nature from the situation.

The writer wishes to express his appreciation to the discussers for bringing out some of the factors which would have to be considered in such a joint venture of Government and private insurance though he does not agree as to the practicability of such action.

Discussion of
"THE BOX INLET DROP SPILLWAY AND ITS OUTLET"

by Fred W. Blaisdell and Charles A. Donnelly
(Proc. Sep. 534)

NEAL E. MINSHALL,[1] A.M. ASCE.—Several variations of the general type of structure reported on by the authors were built quite extensively in southwestern Wisconsin during the period 1933 - 38. As stated by the authors the type tested by them has replaced the ones formerly used and likewise the inlet has now been further modified to overcome design and construction difficulties and affect a material saving.

The writer examined 52 of the original type during a four county survey of structures in Wisconsin in 1949 and reported the results in SCS-TP-116.(11)

While the authors tests were limited to a maximum D/W ratio of 1.0, the field examination showed that one-third of the structures as built in this area had a D/W ratio of about 1.5. Since many of the existing structures might well be used as sites for estimating peak discharges from unusual floods it is unfortunate that complete data are not available for this higher D/W ratio. If the structure is to be used for vertical drops up to 12 feet then if the D/W ratio is limited to 1.0 the structure will normally be wider than necessary. In general the minimum concrete quantity for the box inlet spillway will be obtained with a structure which is no wider at the headwall than necessary to discharge the required Q for the design values of H and D. A wider structure will not permit a significant reduction in total weir length and for a specific D the base area of the inlet will increase as the D/W ratio decreases.

A still different modification of this type inlet eliminates the long headwall and continues the sidewalls up along 3 to 1 fill slopes, using only a moderate length of cutoff under the center of the fill. Tests to determine the hydraulic capacity of this type structure were made, at the writers suggestion, as a thesis study at the University of Wisconsin in 1953(12) under the direction of Dr. Arno T. Lenz. Additional check tests on this type of inlet were made by the writer. This inlet has been designated by the Washington Design Section of the Soil Conservation Service as the trapezoidal weir box inlet. Some of the results obtained on the trapezoidal weir box inlet were presented before the American Society of Agricultural Engineering.(13) Advantages of this type are a saving of material and reduction of scour around the inlet and at the toe of the fill.

For a comparison of inlet types the same model was used for tests on the standard type inlet, Figure 2, (as previously used on the trapezoidal weir box inlet) to determine the effect of dike location on discharge for various ratios of B/W. The Wisconsin series of tests were made for a range of designs as shown in Table 8. Each of these listings represent a series of about fifteen individual test runs. The total number of runs was approximately 1000.

The effect of the dike position on the discharge coefficients for B/W values

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of 1.0 and 0.5 are not the same as assumed by the authors. Comparison of the results in Table 4 with the values obtained in the University of Wisconsin tests are shown in Table 9. The slight disagreement in the correction values for B/W of 2.0 result from the fact that the Wisconsin tests had the height of dike approximately 1.33H to allow for freeboard. Thus the toe of the dike extended upstream a distance of 4H from the headwall rather than 3H. The Wisconsin tests shown in the Table were made for a design H/W of 0.42 whereas those reported in Table 4 were for H/W = 0.35. Thus for X/H of 1.0 the toe of slope in the Wisconsin tests was 0.07W farther from the spillway crest. Only values of the discharge coefficient for X/H of 1.0, 0.5 and 0 were determined in the Wisconsin tests since nearly all field installations have been designed with the toe of the fill back a distance of H from the weir crest. The corrections for X/H = 0.7 in Table 9 were interpolated. These results show that for B/W of zero,which is a straight drop spillway, the effect of the dike is to increase the discharge, the greatest capacity resulting when X/H = 0.5.

The corrections shown in Table 9 apply only at the specific design head H = 0.42W. For heads less than the design head there will be a slight additional reduction in the discharge coefficient as shown in Figure 23.

The effect of box inlet shape B/W on the discharge coefficient, as determined from the Wisconsin tests, show good agreement with the curve of Figure 8. These tests also indicate that the value of $c_2\sqrt{2g}$ in Eq. 3 may be a constant for all ratios of D/W greater than 1.0.

The authors state that "It will be noticed that the discharge coefficient is a constant in Figure 9 when H/W is greater than 0.6." The graph of Figure 6 does not bear out this statement but indicates, for that particular test, a reasonably constant coefficient between the limits of H = 0.05 and 0.15 and a gradual decrease in c_3 for higher heads. In equation 6, which applies to the lower part of the curve of Figure 6, control is at the spillway crest. For convenience of plotting the authors have shown $Q^{2/3}$ versus H which is the same as raising both sides of equation 6 to the two-thirds power.

Thus $Q^{2/3} = 3.43^{2/3}\,L^{2/3}\,H$ or

$Q^{2/3} = KH$; where $K = (3.43L)^{2/3}$

Which can also be written $Y = KX$ where; $Y = Q^{2/3}$; $X = H$; and K is the constant. This is the equation of a straight line of slope K passing through the zero of coordinates. For higher heads the line does not pass through the origin and the coefficient therefore cannot be a constant.

In the Wisconsin tests, for the deepest box of D/W = 1.5, the spillway crest appeared to drown out at H/W about 0.5. During the tests on the trapezoidal weir box inlet, water surface profiles were taken at the toe of the slope of the 3 to 1 dike. One series of these profiles for H/W of about 0.5 and various ratios of B/W are shown in Figure 24. From this it appears that the flow over the sides probably becomes a submerged weir at some point below H/W = 0.5 with a resultant decrease in discharge coefficient for higher heads. Discharge coefficients for the Wisconsin series of tests with headwall similar to Figure 2, without dikes, and for D/W = 1.5 are shown in Figure 25. The significance of these coefficients is that they are reasonably constant for heads between 0.05 and 0.20 while the control remains at the crest. With control at the headwall, which condition begins below H/W = 0.5, for most boxes, the coefficients based on Eq. 6 or Fig. 9 for H/W ratios above 0.5 have little practical significance. The only exception to the results shown in Figure 25 would be for B/W = 0.0, or a straight drop spillway in which the coefficient might remain reasonably constant through a wide range in heads or until submerged by high tailwater.

The discharge over the sloping section of the trapezoidal weir box inlet was determined by plotting discharge against head for this type and for the type shown in Fig. 2 without dikes on the same graph. The difference in discharge at any head was credited directly to the sloping section. These differences in discharge plotted against head were fairly constant for all B/W ratios. This method of direct comparison eliminated the necessity of applying corrections for B/W and H/W ratios to Eq. 6. The flow over the sloping section can be considered as that over a triangular weir in which the discharge is proportional to the five-halves power of the head or

$$Q_s = C_s H^{5/2} \tag{14}$$

Computations for a range of H/W ratios between 0.20 and 0.40 gave an average value of $C_s = 9$. The equation for the trapezoidal weir box inlet without headwalls can therefore be written

$$Q = 3.5LH^{3/2} + 9H^{5/2} \tag{15}$$

In Eq. 15 the only correction which need be made is for the effect of the B/W ratio. Comparison of discharges for the type shown in Fig. 2, with the toe of the dike H distance from the spillway crest, and the trapezoidal weir inlet with sidewalls sloping up along 3:1 fill slopes gave an increase of approximately one-third for most B/W ratios.

The authors state that, "Apparently the toe of the dike should be located from 3H to 5H from the box inlet crest in order to minimize the dike effect" and, "The importance of keeping the toe of the dike well away from the box inlet crest is quite apparent from the values shown therein." These statements would perhaps be true if the main criteria of good design were to keep the crest length L to a minimum. There are many situations where keeping the dike back the recommended distances would require extensive and costly excavation since the headwall is usually carried down 3 feet or more below the spillway floor to act as a cutoff. The differential loading on the long headwalls require they be designed as cantilever retaining walls with footings.

To compare the two types of inlets to determine the difference in concrete volume for the same capacity structure two designs were made for Figure 2 and a third using the trapezoidal weir inlet with the following results. For all three designs Q = 500 cfs, D = 6 ft., W = 7.5 ft., H = 3 ft., and the same outlet. Only the length of level portion of the inlet (B) and length of headwall were variable. For design I with X = 3H, B = 11.5 ft. and the length of headwall is 21 ft. each side. For design II with X = H, B is increased to 12.5 ft., the headwall lengths are reduced to 15 ft. with a resultant saving in concrete of about 8 percent. Moving the toe of the dike closer than H distance from the spillway crest will not result in additional savings in concrete quantities. For design III with the trapezoidal weir box inlet B = 6.5 ft. and with the length of cutoff of 5.0 ft. there is a further saving in concrete volume of 10 percent over design II.

On the basis of this limited analysis, it appears that the most logical design for the type inlet shown in Fig. 2 is with X = H which places the toe of the fill as far from the spillway as possible without increasing the concrete required. Of course as the toe of the dike is moved nearer the spillway crest it may become necessary to rip-rap the end of the fill to prevent excessive scour.

For very large structures further savings might be effected by flaring the

sides uniformly from a minimum W of 2H at the upstream end of the inlet to W_e at the outlet cutoff. This change might require re-evaluation of both the inlet and outlet characteristics.

In all the tests, at both Minnesota and Wisconsin, the approach channel was level with the box inlet crest. Many of the structures examined in the field have built up silting gradients of about one percent above the spillway crest. Therefore on future tests it would be advisable to determine the effect of silting gradients on the discharge capacity.

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Table 8 Effect of Dike Location

(Design H/W values tested)

D/W		B/W 0	B/W 0.5	B/W 1.0	B/W 1.6	B/W 2.0
0.5	X/H = 0	.22 .42	.22	.22	.22	.22
	X/H = 1.0	.22 .42	.22	.22	.22	.22
1.0	X/H = 0	.42 .60	.42 .60	.42 .60	.42 .60	.42 .60
	X/H = .5	.42	.42 .60	.42	.42	.42 .60
	X/H = 1.0	.42	.42 .60	.42 .60	.42	.42 .60
1.5	X/H = .5	.50	.50	.50	.50	.50
	X/H = 1.0	.50	.50	.50	.50	.50

H,B,D,W, and X according to Figure 2.

Table 9 Correction for Dike Effect - Control at Box Inlet Crest

B/W	X/H 0.0		X/H 0.5	X/H 0.7		X/H 1.0
	Table 4[a]	Wis[b]	Wis[b]	Table 4[a]	Wis[c]	Wis[b]
2.0	0.84	0.72	0.82	0.85	0.84	0.86
1.5		0.73	0.85		0.87	0.89
1.0	0.73	0.78	0.92	0.75	0.94	0.96
0.5	0.60	0.93	1.01	0.62	1.01	1.01
0		1.04	1.08		1.06	1.04

a. H/W = 0.35, toe of slope extends upstream 3H or 1.05W

b. H/W = 0.42, toe of slope extends upstream 4H or 1.68W

c. Interpolation

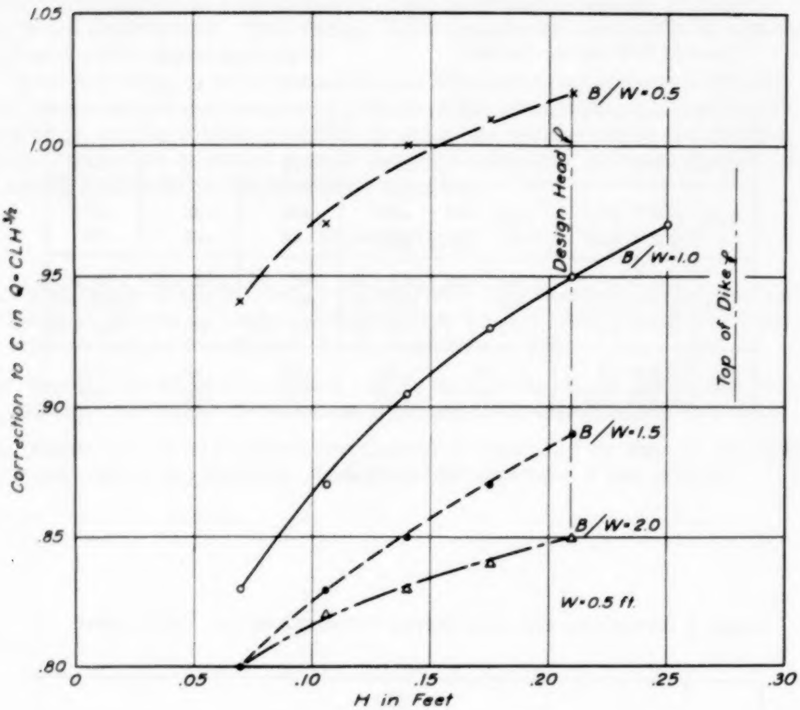

Fig. 23. Correction for Dike Effect for Heads less than Design H.

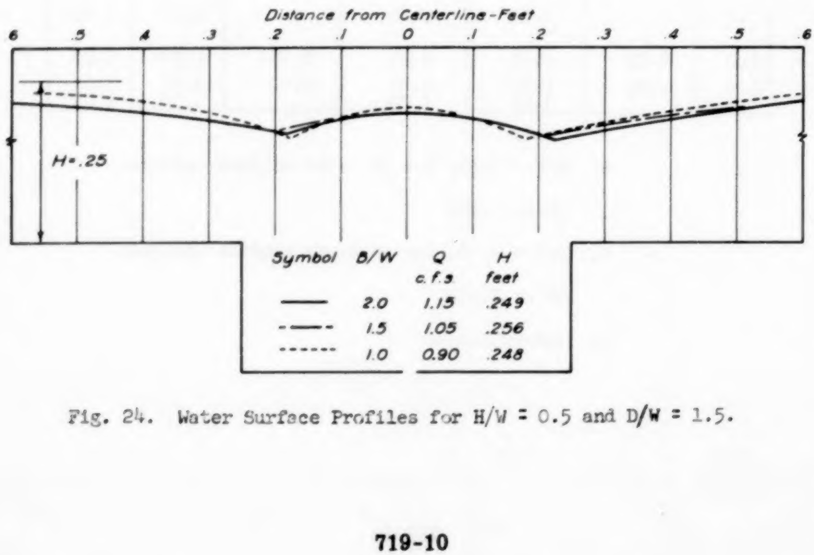

Fig. 24. Water Surface Profiles for H/W = 0.5 and D/W = 1.5.

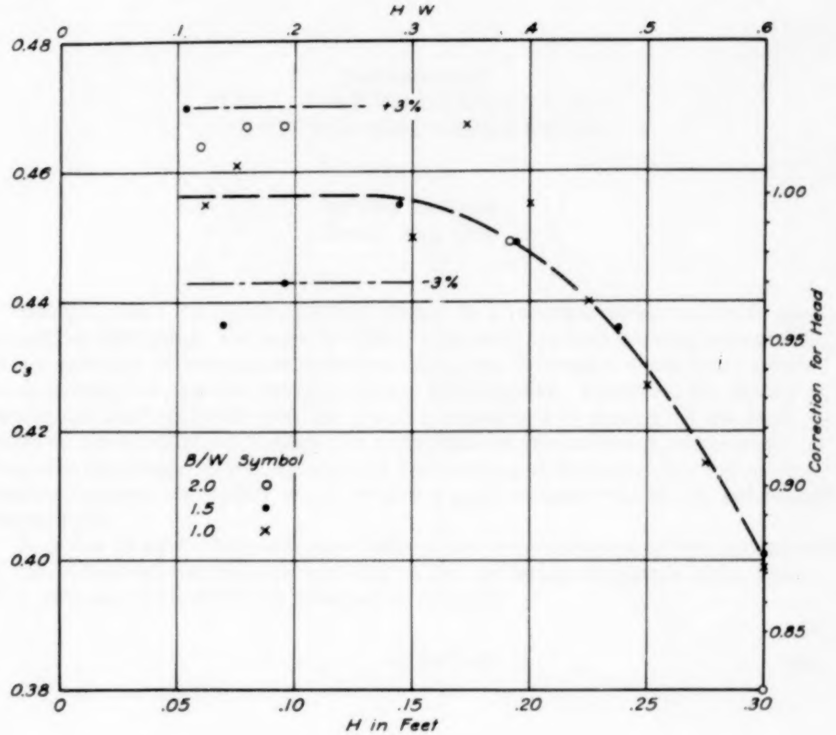

Fig. 25. C_3 in $Q = C_3 L \sqrt{2g}\ H^{3/2}$ corrected for B/W

Discussion of
"THE LOG-PROBABILITY LAW AND
ITS ENGINEERING APPLICATIONS"

by Ven Te Chow
(Proc. Sep. 536)

SHUH-CHAI LEE.[1]—Mr. Chow's paper is a valuable contribution to engineering statistics, because it offers a general method not only applicable to the analysis of various engineering data, but, through a theoretical treatment linking the famous works of Hazen and Gumbel. However, the author seems not well satisfied with his graphic procedure to determine the best value of the coefficient of skew nor with Hazen's correction formula and Foster's diagram. While we wait for the coming of Foster's new set of correction factors, the writer suggests now a method described in the subsequent paragraphs.

In order to avoid the confusion between the nomenclature of the parameters of the statistical distribution and that of the statistics of sample data, Eqs. (11), (14) and (15) should be changed as follows:

$$\mu_x = e^{\mu_y + \sigma_y^2/2} \tag{44}$$

$$\gamma_v = \sigma_x/\mu_x = (e^{\sigma_y^2} - 1)^{1/2} = \phi_v(\sigma_y^2) \ldots\ldots\ldots\ldots \tag{45}$$

$$\gamma_s = \alpha_x/\sigma_x^3 = (e^{\sigma_y^2} + 2)(e^{\sigma_y^2} - 1)^{1/2} = \phi_s(\sigma_y^2) \ldots\ldots\ldots\ldots \tag{46}$$

$$C_v = S_x/\bar{x} = (e^{S_y^2} - 1)^{1/2} = \phi_v(S_y^2) \ldots\ldots\ldots\ldots \tag{47}$$

$$C_s = a_x/S_x^3 = (e^{S_y^2} + 2)(e^{S_y^2} - 1)^{1/2} = \phi_s(S_y^2) \ldots\ldots\ldots\ldots \tag{48}$$

where C_v and C_s, calculated from the observed data $x_1, x_2, \ldots\ldots x_n$ by formulas:

$$C_v = \sqrt{\left[n/(n-1)\right](\overline{x^2}/\bar{x}^2 - 1)} \ldots\ldots\ldots\ldots \tag{49}$$

$$C_s = \left[n/(n-1)\right](\overline{x^3} - 3\bar{x}\,\overline{x^2} + 2\bar{x}^3)/\bar{x}^3 C_v^3) \ldots\ldots\ldots\ldots \tag{50}$$

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in which n is the total number of items, $\bar{x} = \Sigma x/n$, $\overline{x^2} = \Sigma(x^2)/n$ and $\overline{x^3} = \Sigma(x^3)/n$, are the estimate of the population value γ_v, the coefficient of variation, and γ_s, the coefficient of skew. Since γ_v and γ_s are functions of σ_y^2 only, the corresponding value C_v and C_s may be guessed to be the similar functions as shown above, of the sample variance, S_y^2, of a normal variate $y = \log_e x$. When S_y^2 has been computed either from

$$S_y^2 = \log_e(1 + C_v^2) = 2.3026 \log_{10}(1 + C_v^2) \ldots\ldots\ldots\ldots \quad (51)$$

or from

$$S_y^2 = \log_e \left(\sqrt[3]{1 + C_s^2/2 + C_s/2\sqrt{4 + C_s^2}} + \sqrt[3]{1 + C_s^2/2 - C_s/2\sqrt{4 + C_s^2}} - 1 \right) \quad (52)$$

σ_y^2 is represented by

$$\sigma_y^2 = S_y^2(1 \pm \epsilon) = S_y^2 + (\pm\epsilon)\, S_y^2 \ldots\ldots\ldots\ldots \quad (53)$$

in other words, the true value of σ_y^2 is in the neighbourhood of S_y^2 with an error factor $\pm\epsilon$ expressed as the following function:

$$\pm\epsilon = (\sigma_y^2 - S_y^2)/S_y^2 = \sigma_y^2/S_y^2 - 1 = (n - 1)/\chi^2 - 1 \ldots\ldots \quad (54)$$

in which the values of χ^2(chi-square) corresponding to (n - 1) degrees of freedom and an assigned critical probability can be obtained in any textbook of modern statistics.

According to the foregoing statement, the corrections of C_v and C_s or the differences between γ_v and C_v, γ_s and C_s are developed by Tylor's theorem as below:

$$\begin{aligned} d_v = \gamma_v - C_v &= \phi_v(\sigma_y^2) - \phi_v(S_y^2) \\ &= \phi_v\left[S_y^2 + (\pm\epsilon)\, S_y^2\right] - \phi_v(S_y^2) \\ &= (\pm\epsilon)\, S_y^2 e^{S_y^2}/2(e^{S_y^2} - 1)^{1/2} \\ &= \left[(n - 1)/\chi^2 - 1\right] S_y^2 e^{S_y^2}/2(e^{S_y^2} - 1)^{1/2} \ldots\ldots \end{aligned} \quad (55)$$

$$\begin{aligned} d_s = \ _s - C_s &= \phi_s(\sigma_y^2) - \phi_s(S_y^2) \\ &= \phi_s\left[S_y^2 + (\pm\epsilon)\, S_y^2\right] - \phi_s(S_y^2) \\ &= 3(\pm\epsilon)\, S_y^2 e^{2S_y^2}/2(e^{S_y^2} - 1)^{1/2} \\ &= 3\left[(n - 1)/\chi^2 - 1\right] S_y^2 e^{2S_y^2}/2(e^{S_y^2} - 1)^{1/2} \ldots \end{aligned} \quad (56)$$

where the terms higher than second order are omitted. The corrections depend upon the variance of y, which has been obtained previously, the size of sample and a desired confidence probability. For same confidence probability, the correction to C_s is $3e^{S_y^2}$ or approximately, by Eq. (21), $3(\bar{x}/\text{Median of } x)^2$ times of correction to C_v. Besides, Eq. (52) is very cumbersome, we would like to use Eq. (51) to compute S_y^2 not Eq. (52).

So long as the closing date of discussion permits of no delay to make further description of the writer's proposed method, it is hoped that its application to numerical examples is left to interested persons.

Discussion of
"HYDRAULICS OF THE FREE OVERFALL"

by A. Fathy and Mahmoud Shaarawi Amin
(Proc. Sep. 564)

HERMAN J. KOLOSEUS.[1]—The writer would like to take this opportunity to make several suggestions and comments regarding the paper on the Hydraulics of the Free Overfall.

The writer believes that the authors may be defeating the point which they are attempting to prove when they make Eq. 7 applicable to the critical depth position through use of the expression

$$\frac{q^2}{gY^3} = 1 \qquad \text{(A)}$$

The authors developed Eq. 7 on the premises that the magnitude of the velocity varies across a particular section and that the pressure distribution is non-hydrostatic. Eq. A is based on just the opposite conditions; i.e.

1) The pressure distribution is hydrostatic, and

2) The velocity is constant across the section considered.

If one wished to express q as a function of Y on the basis of minimum specific head for the case in which the velocity was not constant across the section and the pressure distribution was non-hydrostatic, then the expression for specific head would have to be revised accordingly.

Quoting the authors (page 5), "The terminal depth is a function of both the discharge and the bed slope." The writer believes that the terminal depth is also a function of the resistance exerted on the flow by the floor of the flume. If such is the case, then this resistance effect might be indicated in Fig. 4 (and for the same reasons in Figs. 5 & 6) if the abscissa were S_o/S_u instead of S_o. S_u as used here refers to the slope for uniform flow.

The writer has found the section concerning negative pressures most interesting even though he finds it difficult to accept the results. In passing, it might be mentioned that Eqs. 11 and 12 which are the basis of Eq. 13 are inconsistent dimensionally and also that the reason for some of the terms is not obvious. Be that as it may, the authors endeavour to support the presence of negative pressure through Eq. 13. This equation is one form of Euler's equation for an inviscid (ideal) fluid and therefore can only be applied with reservation to real fluids. The assumption of an inviscid fluid in the neighborhood of the drop is not greatly in error because the acceleration of the fluid due to the pressure gradient causes the real fluid to behave in a manner similar to that expected of an inviscid fluid. If the fluid is considered to be both inviscid and irrotational with S_o equal to zero, then the problem can be resolved by means of the flow net. An analysis of the flow net given by

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Rouse[1] indicates that the pressures above the drop are greater than atmospheric. This fact is contrary to the results given by the authors in Fig. 7. Actual pressure measurements above the drop by Rouse[1] indicate that the fluid pressures are greater than atmospheric.

The writer appreciates the problem which the authors faced in ascertaining the pressure in curvilinear flow. When measuring pressure in this type of flow, the plate containing the piezometer hole should conform in curvature to that of the streamline at the position at which the pressure is being measured so that the motion of the fluid being measured is disturbed as little as possible. Use of a flat plate to perform this task would disturb the fluid motion and effect the pressure. Also, the presence of the ends of the pressure tubes near the leading edge of the "Direction Vane" may cause zones of separation above the piezometer openings which would result in reduced pressure indications by both piezometers of the "Direction Vane." If the pressure indicated by the "Direction Vane" is a function of the velocity, then one might anticipate the existence of a correlation between velocity and pressure. Since the jet velocity increases with downstream distance, then one might conclude from the data of Fig. 7 that the possibility exists that the "Direction Vane" pressure indications are not independent of the velocity.

The authors could, and possibly have, checked their results by determining the discharge through integration of the velocity across the jet and comparing this figure with the reading from a flow meter. If the integrated discharge were consistently higher than the discharge indicated by the flow meter, then one might suspect the "Direction Vane" of indicating pressures which are low which could account for the excess discharge and the negative pressures within the body of the fluid.

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M. R. CARSTENS,[1] A. M. ASCE, and R. W. CARTER,[2] A.M. ASCE.—Messrs. A. Fathy and Mahmoud Shaarawi Amin have presented new data concerning the characteristics of the free overfall as the channel bottom slope S_o is varied. The purpose of this discussion is to consolidate data from sources that were not mentioned in the paper and to point out several errors in the analysis and results.

The two-dimensional free overfall is one limit of the rectangular suppressed weir, that is, a weir of zero height. Since the rectangular suppressed weir (including the free overfall) is a control section, the flow characteristics are uniquely determined by the geometry, viscous forces, and the capillary forces. For a free overfall in a long uniform channel the geometric variable is simply the bottom slope S_o. The role of the viscous forces is determined by the values of the relative roughness ϵ/Y and the Reynolds number q/ν. The role of the capillary forces is negligible since at no point does the free surface have a small radius of curvature. Therefore, any characteristic such as the ratio of brink depth to critical depth Y_t/Y_c can be indicated in the following manner for the two dimensional free overfall.

$$Y_t/Y_c = \Phi_1(\epsilon/Y,\ q/\nu,\ S_o) \tag{1}$$

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Further simplifications can be made in Eq. (1) by using the energy line slope S_f which incorporates the essential factors which led to the inclusion of ϵ/Y and q/ν. Since the depth of flow Y in the vicinity of the free overfall will always be close to Y_c, the critical slope S_c can be substituted for the energy line slope S_f. Thus,

$$Y_t/Y_c = \Phi_2(S_c, S_o) \tag{2}$$

It would seem possible to further simplify this relationship since a variation in either S_o or S_c would have the same effect on the flow profile. For example, a decrease in the bottom slope S_o would have a similar effect as an increase in the boundary roughness or S_c. A test of this hypothesis is found in the authors' Eq. (7) written in the following form:

$$\frac{dY}{dx} = \frac{S_o - S_f - (Y/2)\,(dY/dx) - (q^2/gY^2)\,(d\beta/dx)}{\alpha - \beta\, q^2/gY^3} \tag{3}$$

α, β, and dY/dx are not independent variables. Therefore, from Eq. (3) and the previous discussion, any characteristic of the free overfall in a long uniform channel is indicated as being a function only of $S_o - S_c$.

$$Y_t/Y_c = \Phi_3\,(S_o - S_c) \tag{4}$$

$$L/Y_c = \Phi_4\,(S_o - S_c) \tag{5}$$

L is the distance upstream from the brink at which the depth of flow Y is equal to the computed critical depth Y_c.

The indicated relationships of Eqs. (4) and (5) are not unlimited. The relative roughness and Reynolds number were initially included to account primarily for the approach-velocity effect. The approach velocity distribution would be the function of these parameters only in a uniform channel section of sufficient length so that the velocity distribution would be determined by boundary shear. An example of a case for which Eqs. (4) and (5) could not be expected to apply would be a broad-crested weir. In the broad-crested weir the water surface profile and the velocity distribution are influenced predominantly by the flow disturbance at the weir entrance with an insufficient uniform channel length for the brink to be in a region of fully developed boundary layer. In this connection, data from the excellent broad-crested weir experiments of Prentice(1) are revealing. From his experiments the average value of Y_t/Y_c of 0.712 was reported which compares favorably with the results obtained in longer uniform channel sections. However, this average was obtained from 24 separate measurements in which variation was from 0.682 to 0.738. Figure 1 is a graph of Prentice's data with the rounded entrance. It is evident from Fig. 1 that the value of L^1/Y_c must be somewhat in excess of 20 before the channel is of sufficient length to consider the free overfall characteristics a function of only $S_o - S_c$.

The relationships indicated by Eqs. (4) and (5) are graphically presented on Figs. (2) and (3), respectively from various sources from which this data could be determined. The most comprehensive study of the free overfall reported was that by Rouse(2) who analytically and experimentally determined the velocity, pressure, and energy distribution in the vicinity of the free

overfall. Moore[3] studied the condition at the base of the free overfall and the datum taken from this source was incidental to Moore's primary study. Southwell and Vaisey[4] determined the potential flow pattern analytically by the relaxation method. Holt[5] measured the distance from the brink at which the depth of flow was equal to the critical depth for various bottom slopes in a two-inch wide channel as an incidental part of a study of flow in highway culverts.

The precision with which the value of Y_t/Y_c (Fig. 2) could be predicted led Rouse[6] to propose the free overfall as an open-channel flow meter which required only the measurement of Y_t for the determination of the discharge. The discharge equation is

$$q = [g^{1/2}/(Y_t/Y_c)^{3/2}]\,Y_t^{3/2} \tag{6}$$

The values of Y_t/Y_c would be determined from Fig. 2 for any given free overfall. Since the value of S_c is fairly insensitive to large changes in the discharge, the value of can be considered to be practically constant for a given free overfall. Nevertheless, the value of $g^{1/2}/(Y_t/Y_c)^{3/2}$ is a function of $S_o - S_c$ and not the constant 1.644 that Rouse presented.

The distance upstream from the brink at which the flow depth is equal to the critical depth (Fig. 3) is significant in the calculation of flow conditions upstream from the brink. At this point the flow depth is equal to the computed critical depth Y_c and the pressure distribution is hydrostatic. If the velocity distribution is also uniform, then the specific energy H_o will be $3Y_c/2$. Therefore, this point is a logical starting point for computation of upstream channel conditions since the depth of flow, pressure distribution, and total energy are known.

The true minimum specific energy section is at the brink and not at the point at which the depth of flow is equal to the critical depth Y_c. The minimum specific energy H_{oc} is a minimum specific energy only if the pressure distribution is hydrostatic (gradually varied or uniform flow zones). In rapidly varying flow zones the pressure distribution is not hydrostatic and no general statement is possible about the value of the true minimum specific energy. In other words, H_{oc} is the minimum specific energy only in uniform or gradually varied flow regions but the true minimum specific energy would always be less than H_{oc} depending upon the boundary conditions. For the free overfall it is possible to estimate the value of the true minimum specific energy H_{ot} by utilizing the data shown on Fig. 3. The parameter $S_o - S_c$ is a measure of the rate at which specific energy is changing with respect to distance in the direction of flow. Therefore, the product of $L(S_o - S_c)$ will be the total change in specific energy in the distance L. On Fig. 4 is shown the value of H_{ot}/Y_c as a function of $S_o - S_c$.

Both in the text and in the graphical presentation of their data, the authors failed to point out that their experiments covered two distinctly different flow regimes. In the region in which $S_o > 0.0024$, the flow approaching the brink was supercritical. In other words, if $S_o - S_c < 0$ the slope is mild, horizontal, or adverse with control at the brink. Conversely, if $S_o - S_c > 0$ the slope is steep and the control has shifted to some upstream point. With the shift in control section the pertinent variables in the problem must also be changed to incorporate the upstream conditions. In fact, if the flow is supercritical the value of Y_c as a variable decreases. Rouse[7] has presented the supercritical flow surface profile characteristics at a brink in the following manner for a horizontal channel.

$$Y/Y_o = \Phi(F_o, x/Y_c)$$

Y_o is the flow depth in the approach channel at a distance which is 3.5 time Y_o upstream from the brink. F_o is the Froude number at this section. The influence of the bottom slope S_o will be of secondary importance in relation to the upstream condition represented by the parameter F_o. Therefore, the author's data for which $S_o > 0.0024$ is not correctly presented as the primary parameter representing the upstream conditions is not included.

Only in case the supercritical flow is uniform approaching the brink can the parameter S_o be utilized to represent upstream conditions. The presence of uniform flow approaching the brink would ordinarily require much greater channel length than would be required for the brink control section (Fig. 1). The flow following an upstream control would be either an S2 or an S3 backwater profile. From a description of the author's equipment, the backwater profile must have been an S2 profile which passed through the critical depth in the vicinity of the inlet. Since the S2 backwater curve approaches the normal (uniform) flow depth asymptotically, uniform flow can never theoretically be attained from this type of upstream control. The length of uniform channel upstream from the brink to attain nearly uniform flow will be a function of S_o, Y_o, and C in which Y_o is the normal or uniform flow depth and C is from the Chezy equation.

Assuming that uniform flow is achieved in a two-dimensional channel the value of Y_t/Y_c can be computed utilizing the profile data presented by Rouse.[6] For this case

$$q = C Y_o^{3/2} S_o^{1/2}$$

From which

$$\frac{Y_c}{Y_o} = \left(\frac{C^2 S_o}{g}\right)^{1/3}$$

Therefore

$$\frac{Y_t}{Y_c} = \frac{Y_t}{Y_o}\left(\frac{C^2 S_o}{g}\right)^{-1/3}$$

Figure 5 is a graphical presentation of the terminal depth to critical depth ratio. The large disagreement between the calculated function and the authors' data is the result of utilizing a channel which is of insufficient length. This conclusion can also be reached by computation of the backwater profile for the value of $S_o = 0.0220$ and for the value of Chezy $C = 129$. At this point, $C^2 S_o/g = 11.4$, the depth approaching the brink is approximately 1.4 times the uniform flow depth.

The majority of evidence is indicative that the pressure is not subatmospheric in the nappe at any level in the brink section. That the authors experimentally found these subatmospheric pressures was probably due to the difficulty of measuring pressures in the interior of a curvilinear flow zone. Theoretically, a piezometer in an interior flow zone must be infinitely thin and must have the curvature of the streamlines at that point. Consequently, any such instrument must be shaped according to the streamline curvature which varies from point-to-point in the brink section. The most trustworthy

method of measuring pressures in this brink section is by means of wall piezometers.

Utilizing wall piezometers, Rouse[2] measured the pressures at numerous points. His experimentally determined points are shown of Fig. 6 and are always greater than atmospheric. The authors' values of $(P/\gamma)/Y_c$ are always less than Rouse's values. Since the more accurate wall piezometers were utilized by Rouse the conclusion is that the movable piezometer employed by the authors tended to underregister the value of the pressure. This same conclusion is varified in the authors' Fig. 7 in which negative pressures are shown in the free falling nappe for which the pressures would be practically atmospheric.

The writers have derived the pressure and velocity distribution from the potential pattern of irrotational flow. The free surface profile of both the upper and lower free surface were taken from the data presented by Rouse.[7] A preliminary stream potential pattern was determined by means of a General Electric Analog Field Plotter. This preliminary stream potential pattern was then refined by relaxation methods. From the refined stream potential pattern the velocity components, the velocity, and the pressure were computed. The computations were performed in three vertical sections, at the brink, at the upstream section $x/Y_c = -0.5$, and at the section $x/Y_c = -1.0$. The computed pressure distributions are shown on Fig. 6. From the computed velocity distributions, the values of the momentum correction coefficient β were computed. The value of β was 1.00^+ at sections $x/Y_c = -0.5$ and $x/Y_c = -1.0$ and increased to the value 1.02 at the brink section. The value of $d\beta/dx$ will always be positive for potential flow at the brink. A negative value of $d\beta/dx$ would indicate that the action of shear forces on the velocity distribution predominates in the vicinity of the brink. Conversely a positive value of would indicate that the action of the inertia forces on the velocity distribution predominates in the vicinity of the brink. The authors' statement that $d\beta/dx$ must be negative in the vicinity of the brink is conjecture.

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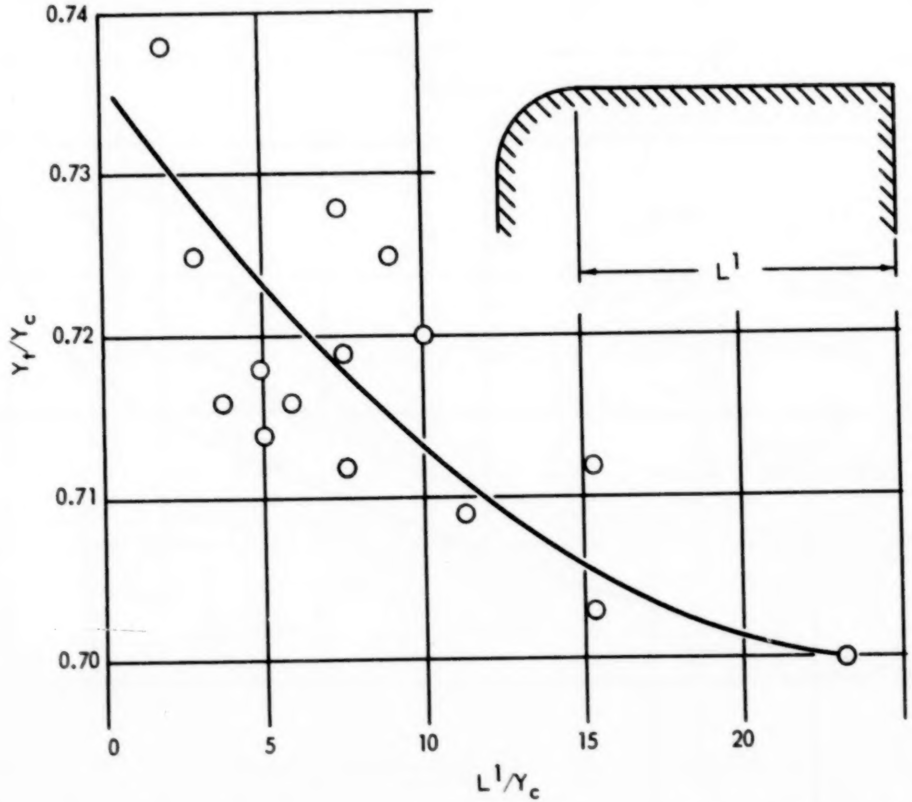

Figure 1. Effect of Uniform Channel Length on the Value of Y_t/Y_c After Prentice [1].

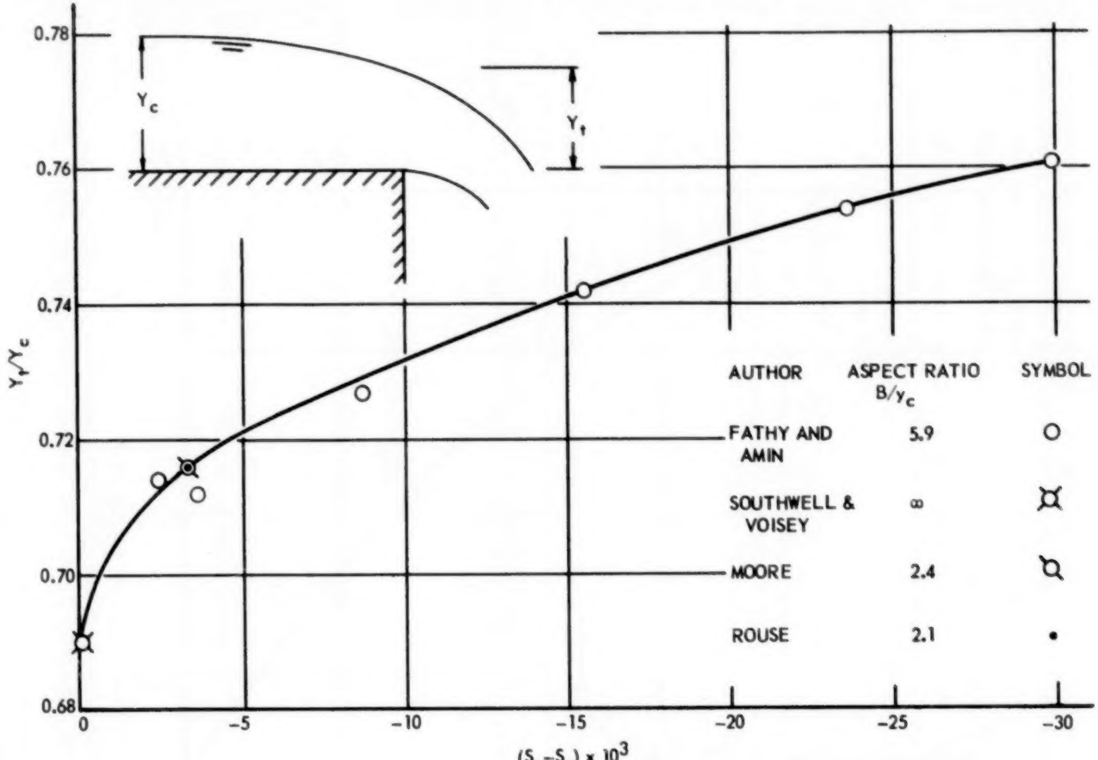

Figure 2. Brink Depth Characteristics of the Free Overfall.

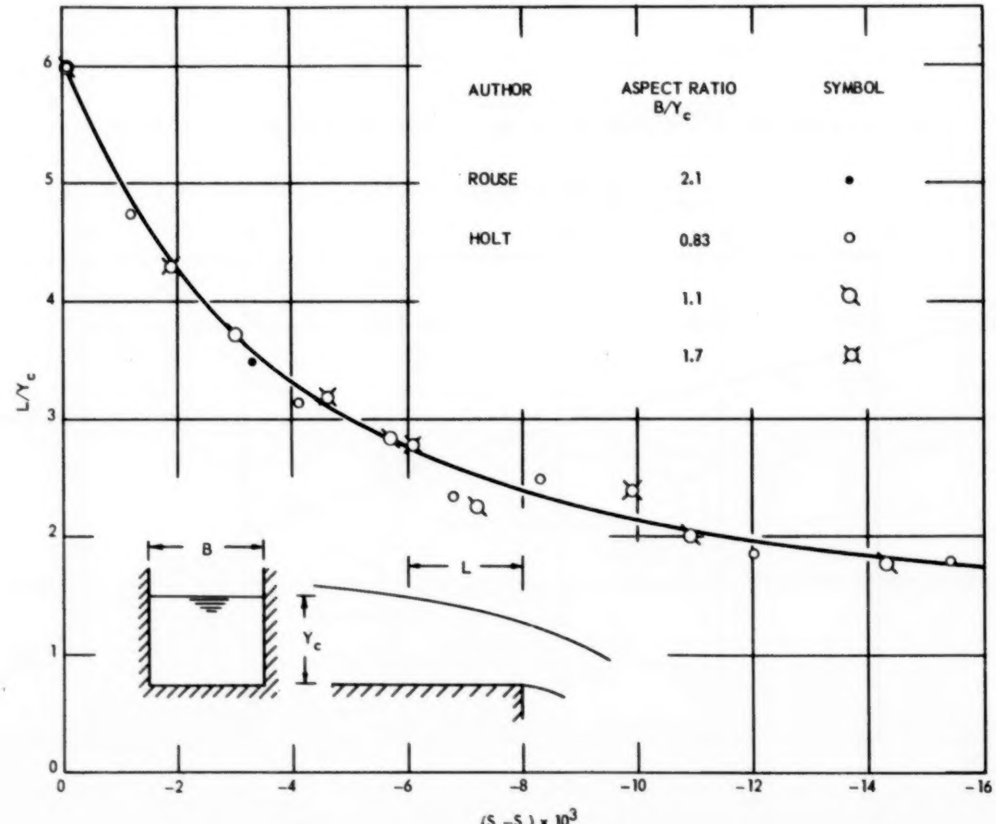

Figure 3. Brink Length Characteristics of the Free Overfall.

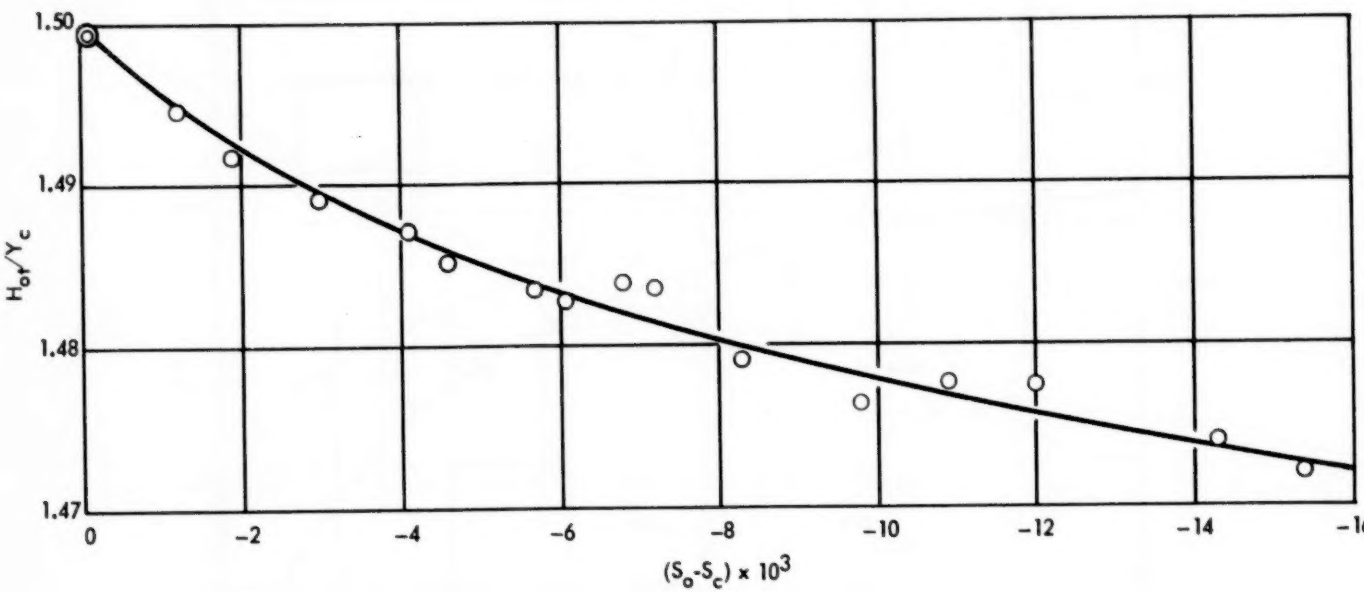

Figure 4. Specific Energy at the Brink Section of the Free Overfall.

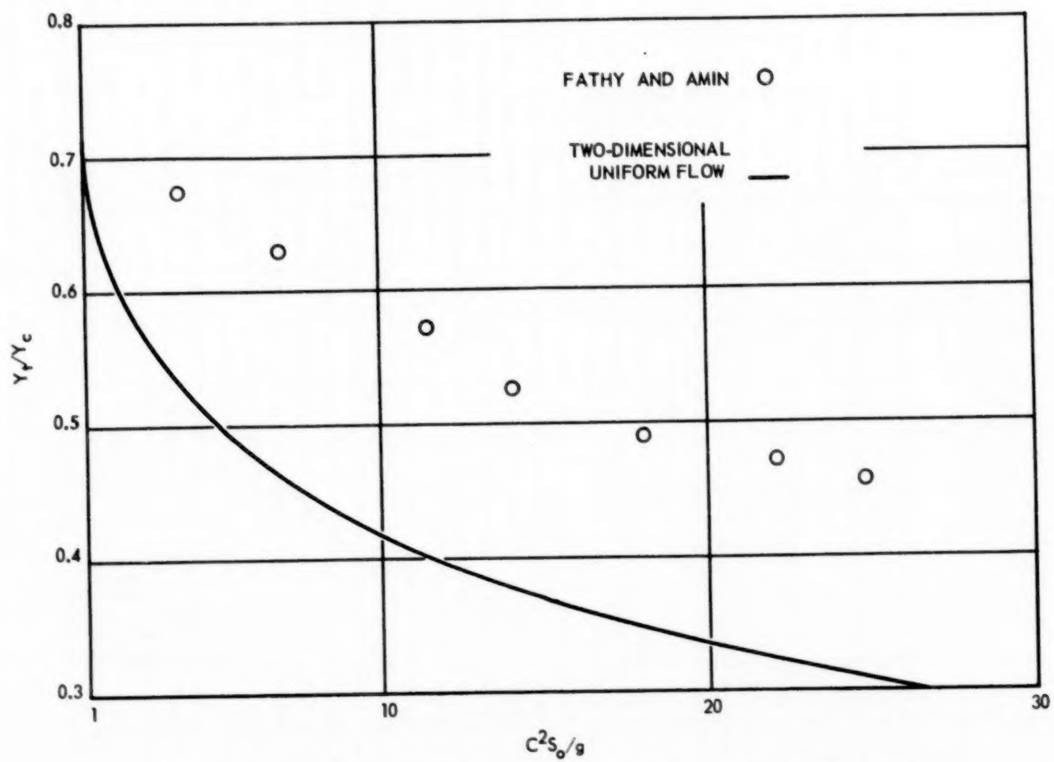

Figure 5. Brink Depth Characteristics with Uniform Supercritical Flow.

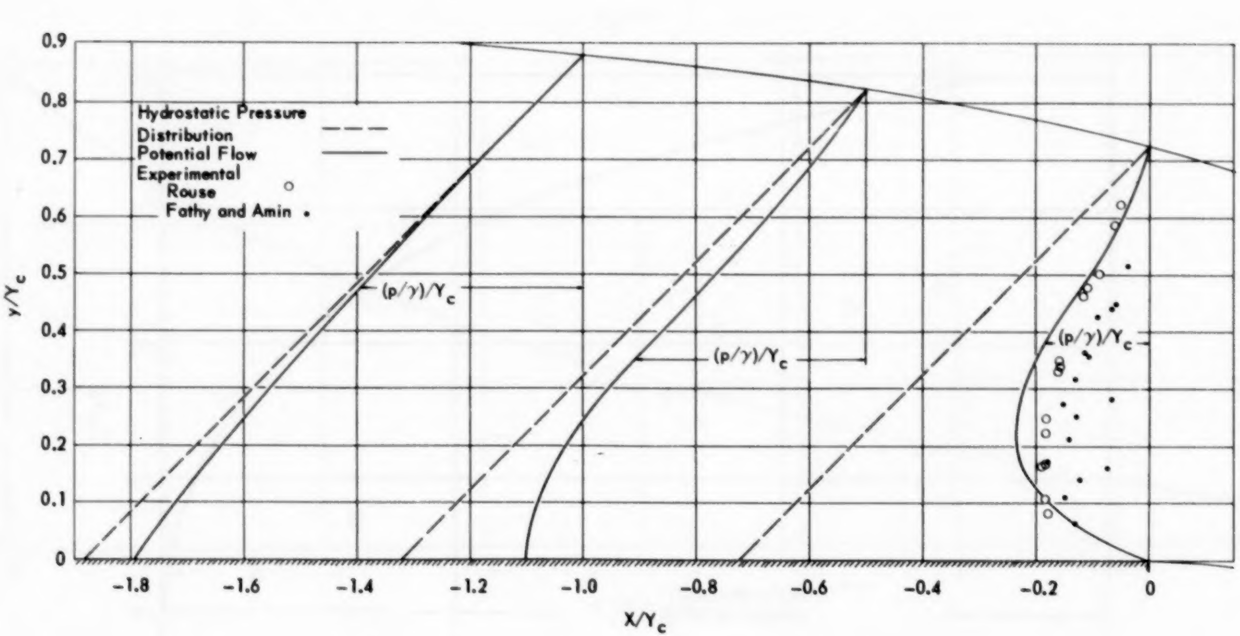

Figure 6. Pressure Distribution at the Free Overfall.

Discussion of
"THE PRESENT STATUS OF RESEARCH ON SEDIMENT TRANSPORT"

by Ning Chien
(Proc. Sep. 565)

T. BLENCH,[1] M. ASCE.—The knowledge outlined by Mr. Chien is almost entirely from laboratory flumes. The various formulas produced, with the mixture of assumptions, approximations, and ingenious ideas behind them are very well presented; the great complexity of the more exhaustive methods is manifest. The vast amount of observation on canals in the field, the dynamical aspect of the formulas evolved therefrom, and the fact that these formulas give a simple and adequate means of practical design that has been used widely for many years, all receive no discussion, though the reference list (74, 76, 77) shows that the information was available for study. Instead, a Figure 6 is proferred, apparently based on laboratory flume experiments with trifling discharges, uniform sediments, and very poorly measurable slopes, to replace the undiscussed formulas that represent the observed self-adjustment of field channels with discharges up to 10,000 cfs, naturally graded bed material, and excellently observeable slopes. As this Figure deals with channels with "bank friction neglected" it cannot be tested as rigorously as a chart for real channels and, of course, as bed-load cannot be measured in the field its prophesies about transport cannot be checked at all, except on a laboratory scale. However, it seems reasonable to assume that the results are intended to approximate those for hydraulically smooth sides. Making that assumption, loci of equal charge ("charge" being measured by ratio of sediment load weight per second to water weight per second), being also loci of equal bed-factor (78, A), would have the properties that the channel slope and depth along any one would vary as the inverse cube root and the cube root of the square of the discharge intensity respectively. (Charge intensity is measured by the cfs. per ft). One point on such a locus is where the 5.0 cfs/ft and the 0.10 tons/day/ft lines cut; another on the same locus is where the 50 and 1.0 lines cut. The former point gives 0.0003 slope and about 3.5 feet depth. Using the law of variation just stated, the latter should give 0.00014 slope and 16.4 feet depth; but it shows 0.0001 and 20 feet instead. This is bad enough, practically; but even worse is that regime formulas (which represent what real channels actually do) give 13.6 feet of depth for 0.25 mm sand, negligible charge, and 50 cfs/ft, and give less depth as charge increases. The writer has had sufficient experience of just that set of conditions to know that 13.6 feet is exceedingly close to nature, and 20 feet has no prospect of realisation as an equilibrium condition.

Incidentally, the one formula quoted from original regime theory, viz equation (39), was investigated in the field by several Indian Provincial Irrigation Research Organisations, who all found that the "constant" depended on the nature of the bed-material. (However, the functional form of the original

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Lacey relations has never had to suffer amendment.) The writer used this information to replace the constant by a simple function of bed and side factors, and published his method of doing so in 1941;(B) the results are in several subsequent references (78, A). This matter is recorded because the author might be thought to have attributed the discovery to Leopold and Maddock in 1953 (75).

The writer regards the laboratory and field approaches to sediment transport problems as equally valid, complementary to each other, but from opposite directions. His comments are intended, not to detract from the former, but to warn against injudicious extrapolation from its results; they are intended also to draw attention to the existence and status of the complementary approach. As there is a readily available summary of regime theory,(A) and a paper due out shortly on its practical use,(C) no details need be given here. Methods of applying regime theory to laboratory data are under present study,(D, E) and are showing the inadequacy of the data for extrapolation.

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JAN M. JORDAAN.[1]—The author, in the section entitled "Density Current," emphasizes the need for a practical approach to determine the usefulness of density currents for conducting silt through reservoirs.

The writer agrees that, in his opinion, the practical considerations are as important as the theoretical in an analysis of this nature. Irregularity of frequency and magnitude of inflows, variation in reservoir levels, temperature and density distributions, slopes and cross-sections may cause the most painstaking analytical treatment to be misleading and of little value where the practical approach is incorrect. Sedimentation in reservoirs is an irregular process in which the greatest damage is done at rare intervals during abnormal floods.

The application of the regime theory to channels may be extended to regime establishment under backwater conditions. In a reservoir the cross-sections in the headwaters progressively become shallower as the sedimentation continues. Stored water, distributed over a large area, becomes subject to increased evaporation losses. The loss in storage capacity, which may be regained by raising the dam, is not as important a factor in limiting the useful

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life of a reservoir, as the increased evaporation losses accompanying the flattening of the cross-sections.

In the writer's opinion, suitable outlet works may be designed for a particular site to release sediment-carrying density currents by the following method of operation:

1) During floods the clear water in the reservoir may be retained long enough for the peak of the turbulent inflow, carrying the most sediment, to be passed as a density current.
2) Release of the suspended silt at the dam, under flood conditions, could be effected by a curtain-wall structure built in front of the spillway crest and extending downward to restrict the region of drawoff to the denser bottom layers, progressing as a density underflow. Silt-bearing water would then be conducted vertically upward against the dam face in the draught-conduit thus created and would spill freely under normal crest head.

It should be borne in mind that such outlet works in the prototype are only useful during rare intervals of flood. Effective operation, however, would increase the useful life of the reservoir, but would have to be verified by model analysis, with due consideration given to accurate representation of the hydrologic features of the actual site.

J. M. LARA,[1] J.M. ASCE.—Mr. Ning Chien has presented an excellent paper on the subject of the present status of sediment research. The paper meets its objective of acquainting the "non-sediment" technologist with the progress that has been made in this field. It is realized that for such a broad subject, it would be quite difficult to expand on the many phases. Mr. Chien, however, has discussed briefly some of the methods currently being used in the computation of sediment transport. Needless to say, there are other research studies being conducted at present to modify or improve the methods. The paper also contains a handy set of references for those desiring more details of the technical background.

Mr. Odin Hanson of the Bureau of Reclamation has prepared a set of curves(7) which show a multiplotting of the SR values vs the unit sediment load as computed by the formulas of Einstein, Meyer-Peter, Kalinske, Straub, and Schoklitsch. Figure 1 shows a graph excerpted from a set of these curves for a 0.125 grain size. From this plotting, it is indicated that further research is needed in an effort to reconcile the highly divergent results of the bed load formulas to within a comparable range of practical limits. It might be attained by the further definition of the existing parameters, empirical coefficients, and other variables and constants presently contained in the different bed load formulas. Progress to this end is being made by the experiments and studies of Chien, Einstein, and the other relatively few sedimentologists.

For current sediment studies, it is necessary for the practical engineer to rely very much on his judgment and experience in making a selection of the proper bed load equation for the problem under consideration. This can be accomplished by making a few preliminary analyses of the various formulas he thinks will be applicable. Hanson's curves can be used as a guide in making the initial selection, providing the engineer is sufficiently acquainted

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with underlying theory of each formula and understands its limits of practical application.

It seems that one of the major pitfalls of the formulas developed by flume experiments is the failure in their application to river channels to obtain comparable results for the same variables which are empirically related by theory or experiment. These formulas may give a reliable answer on one river system, but practically fail when applied to another system. It is the trend of late to bring about a better correlative agreement of the results of the model to prototype. Studies of Pemberton,(1) Colby and Hembree,(2), Chien and Einstein,(3) show a step in this direction. Many of the practical aspects are being considered and a more thorough analysis is being made of each of the natural influential or dominating factors. Mr. Chien pointed out examples of these in the studies being made of Von Karman's k constant and the z exponent of the suspended load theory. These are some of the important steps in bringing closer harmony between the classical theories of experimental verification and practical results. It would be very favorable to continue further research studies to keep these two conditions abreast of each other.

The study of Colby and Hembree(2) merits further discussion to illustrate a realistic approach to research work. The objective of the study was to check the various bed load formulas by using the data collected at a test reach on the Niobrara River near Cody, Nebraska. Of all the formulas tried, it was found that the computed values using the Einstein method gave the best results when compared to the observed sediment discharge. In a further analysis, however, another method was developed for computing the sediment load. The method called the modified Einstein procedure is based on Einstein's theory except that different methods of computation are used to determine (a) the exponential measure of z, (b) the shear velocity with respect to the sediment particles, u_*, and (c) the intensity of bedload transport ϕ_*. The z values are determined by a trial and error solution of the equation

$$(PI_1'' + I_2'')\left[\frac{PJ_1' + J_2'}{PJ_1'' + J_2''}\right]$$

and using the ratio of fall velocities. The shear velocity with respect to the sediment particles is also determined by a trial and error procedure to solve the modified equation (see Einstein(9) p. 10).

$$\bar{u} = 5.75 \left[g\,(SR)_m\right]^{\frac{1}{2}} \log_{10} \frac{12.27\ dx}{k_s}$$

where $\bar{u}$ is the average velocity for the cross section taken from a streamflow measurement.

$(SR)_m$ is the quantity that is obtained by solving the above equation knowing the $\bar{u}$ value and assuming x.

d is the average depth of the cross section rather than R' as given by Einstein.

x is the parameter for transition of smooth-rough boundaries and is an indirect function of the shear velocity, u_*.

u_* results from the evaluation of the terms under the radical sign $\sqrt{g\ SR_m}$.

The intensity of the bed-load transport, ϕ_* is read from Einstein's graph ϕ_*

vs ψ_*. The appropriate value for ψ_*, however, is selected from solving the two equations

$$\psi_m = \frac{1.65\ D_{35}}{(SR)_m}$$

and

$$\psi_m = \frac{0.66\ D}{(SR)_m}$$

The equation yielding the larger ψ_m is used.

The major advantage of the procedure is that it greatly reduces the necessary field work that is required for the original Einstein formula. Information is collected at only one cross section, and neither point-integrated samples nor water-surface slopes are required. The procedure is also closely tied to more of the measured field data.

The writer wishes to add to this discussion, the need that exists today for two more types of research work: (1) the determination of loads carried in cobble-bottom channels, and (2) a method for computing the degradation below dams. These two problems are commonly encountered and are highly important to the practical engineer. To the writer's knowledge, there are no other studies or development of methods to include sediments coarser than the 30 mm size, the upper limit of the Meyer-Peter formula mentioned by Mr. Chien. Computations using any of the prevailing methods have not always given favorable results in the cobble-bed streams. Research in the further experimentation or modification of present or newer methods is definitely needed. Upon checking the available literature on sediment research, the writer could find no methods yet developed for computing degradation below dams. There are studies[4] where the amount of degradation was measured or observed subsequent to the construction of the dam. Miller[5] has investigated some of the practical methods of approach for the computation of the probable degradation which bear mentioning. His analyses involve the use of (a) tractive force principle, (b) competent bottom velocity using the curve developed by Mavis and Laushey,[6] (c) bed-load transport formulas utilizing Hanson's curves,[7] and (d) canal stability which involves Hjulstrom's chart.[8] The procedure essentially consists of plotting the discharge vs the movable size material and checking the size analysis curve for the degradation limit of the channel being considered. Upon establishing a reasonable estimate of the degradation, it is combined with the tail-water rating curve for use in designing the stilling pool.

It is hoped that this discussion will serve to stimulate further research work in the two problems just mentioned and to continue present research studies along the other practical lines suggested earlier in this report.

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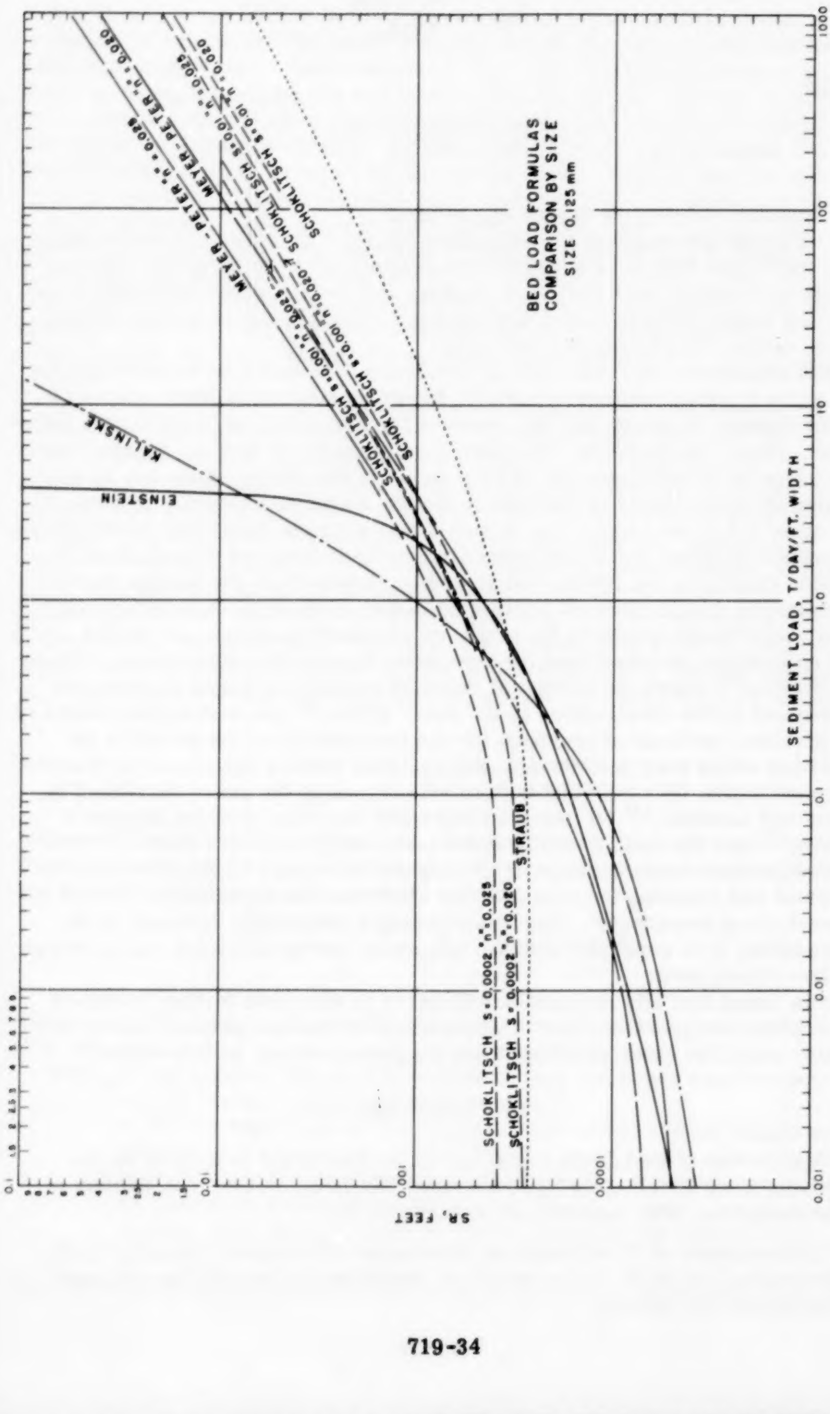
BED LOAD FORMULAS
COMPARISON BY SIZE
SIZE 0.125 mm
SEDIMENT LOAD, T/DAY/FT. WIDTH
SR, FEET
MEYER-PETER n'=0.020
MEYER-PETER n'=0.025
SCHOKLITSCH s=0.01 n'=0.020
SCHOKLITSCH s=0.01 n'=0.025
SCHOKLITSCH s=0.001 n'=0.020
SCHOKLITSCH s=0.001 n'=0.025
KALINSKE
EINSTEIN
SCHOKLITSCH s=0.0002 n'=0.025
SCHOKLITSCH s=0.0002 n'=0.020
STRAUB

Figure 1

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NAVINCHANDRA L. RUPANI, J.M. ASCE,[1] and JOHN B. STALL.[2] Research workers and engineers in the field of sediment transportation will welcome this stimulating and timely article by Mr. Ning Chien, A.M. ASCE.

Grain Resistance

Suspended sediment transportation theory requires essentially the understanding and application of laws governing the velocity distribution in a vertical. Von Karman's velocity deficiency law applied to circular pipes is given by equation (3). The numerical value of Von Karman's universal constant as evaluated from Nikuradse's experiments is found to be 0.4 for clear water flow in circular pipes. That the velocity distribution in a vertical for a sediment-laden flow is different from that of a clear water flow is demonstrated by the variation in the universal constant k from 0.2 to 0.4. There appear to be two schools of thought as to the probable cause of change in the k value. One subscribes to the belief that the sediment in suspension causes dampening of the turbulent eddies and the other attributes the change in k value to the bed roughness. With due respect to the latter the writers favor the former school. Undoubtedly when the bed protrusions caused by the sediment become comparable to the depth of flow the velocity distribution may be expected to change; however, it is not clear how the problem of suspended sediment is affected by such bed-roughness. To begin with there must be sufficient sediment grains in the bed to go into suspension. The space between the protrusions may then be expected to be filled in by these relatively fine particles, thus reducing the ultimate effect of the protrusion height as a rough surface. If, on the other hand, relatively fine sediment grains do not exist in the bed then the problem of suspension does not arise.

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The author's arbitrary division of sediment-laden flow into a heavy fluid zone and a light fluid zone indicates the dampening effect of the highly concentrated zone upon the turbulence generated at the bed. Kalinske and Hsia,[19] using particles having a median diameter of 0.011 mm, found the suspended sediment distribution uniform throughout the vertical with concentrations as high as 11 per cent. Whether the reduction in k value can be similarly explained by dampening effect is not understood.

The relationships between parameter $\sum \frac{C_s V_s}{VS_e} \frac{\rho_s - \rho_f}{\rho_s}$ and k, remembering that the scatter is wide even when plotted on semi-log paper, are of considerable interest. If the wash load with extremely high concentrations exerts any significant influence upon the dampening of turbulent eddies, the relationships as shown in Figure 1 may be expected to change.

Bar Resistance

When evaluating the frictional component due to channel irregularities one must resort to such functional relationships as

$$\psi' = \frac{\rho_s - \rho_f}{\rho_f} \frac{D_{35}}{R'_b S_e} \quad \text{vs} \quad \frac{\bar{U}}{U''_*}$$

developed by Einstein[3] from field measurements. Lane[A] observes that "if a channel is supplied with a heavy bed load, in order to be stable it must move this load along. To be stable, the channel carrying bed loads, therefore, should have a higher velocity along the bed, but the same velocity along the banks, and this could only occur with a wider, shallower section." Griffith[B] also observes similar phenomena in India. In these channels evaluation of bar resistance becomes important. Preliminary calculations, representing Illinois streams and sediment characteristics, indicate ψ' value as low as 0.05 for an average depth of 7 feet. Einstein[3] gives the lowest ψ' value as 0.48 based upon river measurements. Field measurements for rivers with fine sediment loads, relatively flat slopes and shallow depths could be of great value to field engineers in properly evaluating bar resistance. Laboratory flume studies do not give satisfactory results because of the sidewall influence upon the flow which tends to straighten out the channel irregularities.[3]

Wash Load

Average grain-size distribution of sediment from eight reservoirs in central and southern Illinois show that 90 per cent of the sediment by weight is finer than 40 microns (0.04 mm). Thus sedimentation of reservoirs in Illinois is a problem of suspended sediment transportation. This confirms the statement by Mr. Chien that " . . . wash load . . . contributes the bulk of the deposition in lakes and reservoirs."

In an earlier article Einstein and Chien showed[65] quite conclusively the impracticability of predicting the wash load from the hydraulic characteristics of the river channels. In the present article the author suggests further work in establishing quantitative relationships between wash load and watershed factors. Anderson[66] has pioneered in this field. Glymph[C] has

reported and compared a number of other fundamental works attempting to establish such relationships. Attempts are underway in Illinois to improve this quantitative relationship.

As a stream channel develops over a geologic era of time the composition of the bed material becomes a result of the discriminatory transport of the parent materials by the flowing stream. The parent materials within the drainage area above the channel may be heterogeneous and thus confound efforts to understand sediment transport by studies of the bed composition and the hydraulic characteristics of the channel.

Suppose the wash load were traced back upstream to the microchannel into which water first collects after falling on the soil. Horton[D] has contributed fundamental thinking regarding these soil channels. The recent work of Ellison[E] is also of basic importance. The bed composition of this small rill of micro-channel could consist of soil particles having a known size-distribution and the characteristics of this channel could be determined.

In this rill the discrimination of the water flow toward the soil particles in movement has operated to a much lesser degree than in a downstream river channel. The material moved in this rill would have a composition much more nearly identical to the bed material than would be the case downstream where the river has collected flow from widely heterogeneous soils and channels. The close identity of the sediment load and the bed material in the rill or micro-channel offers the possibility of a better understanding of the true sediment transport function which relates the former to the latter.

The sediment load moved in this micro-channel later constitutes a fraction of the total sediment load of a downstream channel although the mode of transport of the different components of the load at the downstream location may not yet be readily classified as bed-material load or wash load. Research into the movement of the load in this micro-channel might reveal the laws which it follows. This can be studied in the laboratory; the model, however, may be larger than the prototype. If some new functions could explain sediment movement in this micro-channel, then this load could be traced downstream to the point where it becomes wash load. It appears that research to better understand the upper networks of the stream systems is requisite to the control of the damaging wash load.

Channel Stability

The problem of stability of river channels and earthen canals deserves a more rigorous approach based upon the modern theory of fluid mechanics. It is probably an oversimplification to postulate that such a simple relationship as $P = 2.67 Q^{1/2}$ developed by Lacey, governs the hydraulics of a stable channel. Indian regime theory based upon the so-called elementary law of silt transportation, formulated by Mr. Kennedy as

$$V = 0.84\, d^{k}$$

lacks satisfactory scientific explanation as to how this "law" was derived. The channels with which Mr. Kennedy worked were assumed to have vertical banks and thus an average depth "d" was used. Average velocity V was obtained by simply dividing the discharge by the cross sectional area. Whether velocity thus obtained truly represents an average over a vertical is debatable since equation (2) is found to be more precisely applicable in describing the average velocity over a vertical. Nothing is known as to what factors govern

the value of constant 0.84. Lacey's[76] empirical formula

$$P = 2.67\,Q^{1/2}$$

though applicable to local conditions in India, has shown a variation of the value of constant from 2.32 to 3.1.[F]

No satisfactory explanation as to what factors govern this variation is available. The writers believe that the following factors, among others, affect the value of the constant.

a) Shape of the channel section

b) Hydraulics of the channel (roughness, energy slope, etc.)

c) Sediment characteristics.

Mr. Lacey also states that the exponent is invariably 0.5. Leopold and Maddock[75] investigated a large variety of rivers in the Great Plains and the southwest regions of the U.S.A. They found an average value of 0.26 for natural streams instead of 0.5 which was strictly obtained from data on stable (regime) irrigation canals in India.

The modern concept of sediment transportation based upon the theory of fluid mechanics developed by W. Schmidt,[9] M. P. O'Brien, M. ASCE,[H] and H. A. Einstein, M. ASCE[3] provides a valid approach to the solution of this difficult problem of sediment transportation. It is gratifying to learn from the author's paper that Einstein's bed-load function gives results which agree with those obtained when proper "silt factors" are used in Lacey's regime theory. It is hoped, as more field experience is gathered covering wide ranges, that the Einstein bed-load function will replace existing empirical relationships, thus providing a universal solution to the difficult problem of sediment transportation in open channel flows.

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